DYNAMIC BEHAVIOR OF A SEISMIC ISOLATED STRUCTURE IN GREECE

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ABSTRACT:

Onassis House of Letters and Arts is a cultural center with hallmark architecture. In order to satisfy the architectural requirements as well as the high performance seismic specifications that were set, a seismic isolation system of FPS type bearings was employed. This study focuses on the methodology followed for the seismic design which, performed in three consecutive steps, includes simple calculations using an equivalent SDOF model, dynamic response spectrum analyses and non-linear time-history analyses using selected earthquake records on a 3-D finite element model. Results indicate good agreement between different analyses and shed light to the behavior of a complex, seismic isolated structure under seismic loads.

KEYWORDS: Seismic Isolation, Friction Pendulum System, Nonlinear Time-History Analysis

1. INTRODUCTION

Onassis House of Letters and Arts is a cultural center with hallmark architecture. Structural design had to meet special architectural requirements -due to the unique shape and function of the building- as well as high performance seismic specifications. Consequently, it was decided to incorporate a seismic isolation system in order to achieve operational performance level under the design earthquake, protection of the valuable contents of the structure and continuation of its functionality after a seismic event. Friction-pendulum (FPS) type bearings were placed under the ground floor slab. Analyses include simple calculations using an equivalent SDOF system, dynamic response spectrum analysis on a 3D finite element model and nonlinear time-history analysis using selected earthquake records.

The study focuses on the methodology followed for the analysis and the design of the structure. From the simplest approach -analysis using a SDOF system- to the more sophisticated one -nonlinear time-history analysis- results show good agreement allowing for a better insight on the dynamic behavior of the building to be gradually gained. Comparison of this behavior to that of the structure if there was no seismic isolation, highlights its importance in similar projects.

2. BUILDING DESCRIPTION

The structure is located in the Athens downtown area. The plan view is orthogonal, with dimensions of 28 m × 66 m. In regards to both its shape and function, the building is divided into two sections, substructure and superstructure.

The substructure consists of 9 floors housing mechanical plant rooms, parking and storage spaces. At the parking areas, slabs are inclined to enable traffic flow. Access to these areas is provided by two independent circular ramps supported by an internal and an external circular wall.

Superstructure (Fig. 4) has a total of 7 levels. It consists of the main section, which is an oval shaped shell structure surrounded by slender columns. An auditorium and a conference hall are located inside the shell with office spaces, libraries, a restaurant, a foyer, venues and lobbies accommodated on the perimeter. Moreover, an exhibition hall, a professional recording studio are also housed, as well as an open-air restaurant capable of hosting art performances which is located on the roof. The overall height of the superstructure is 27 m.

3. SEISMIC DESIGN REQUIREMENTS

High performance seismic specifications were set by the owner. Thus, while Athens, according to Greek Seismic Code 2000 (GSC-2000) -similar to EC8-, is in zone I with a PGA of 0.16g, a value of PGA = 0.24g, corresponding to zone II, was used -predicting an increase in PGA values in a future update of the code. In addition, the importance factor was taken as equal to $\gamma_I = 1.15$ which in fact results to a bigger return period for the design earthquake, normally considered as 475 years. Finally a value of $q = 1.5$, corresponding to elastic behavior, was assigned to the behavior factor in order to minimize damages both to structural and non-structural elements. The architectural concept -the need for small size columns on the perimeter to permit a clear view of the egg-shaped interior shell and the presence of marble facades- could not be met with these high-performance seismic specifications. Thus the incorporation of a seismic isolation system was found necessary in order to achieve: operational performance level, protection of the contents of the structure (i.e. artifacts, recording equipment etc) and continuation of its functionality in case of a moderate to strong earthquake.

Regarding the installation of seismic isolation two possible solutions were examined: installation at the foundation level (-27.70 m) or installation at the ground floor level (-1.60 m). For reasons of reduced cost and construction effectiveness the first solution was rejected since it would lead to: (a) increased vertical loads on the isolators, (b) need for a seismic gap in all 9 underground floors (c) difficult access for inspection and replacement of bearings. Supporting the decision of installing seismic isolation at the ground floor level the following were considered: (a) the low seismic action imposed to the basement –a rigid and non-deformable structure- (b) the thick (1.50 m) ground floor slab which is required irregardless, due to the fact that the grid of the vertical elements of the basement did not coincide with the corresponding grid of the superstructure.

Seismic demand was calculated according to GSC-2000. However since the code has no provisions at this time for seismic isolated buildings, provisions from NEHRP and SEAOC were used.

4. ANALYSIS OF AN EQUIVALENT SDOF SYSTEM

4.1. Dynamic properties of a SDOF system based on a FPS type isolator

The structure can be modeled as a SDOF system of mass, m, based on an FPS type isolator. The main characteristics of the isolator are the radius of curvature of the sliding surface, R, and the friction coefficient, μ. The behavior of the system is described by the bilinear model of Fig. 1. Effective stiffness, K_{eff}, is:

$$K_{eff} = \frac{N_{sd}}{R} + \frac{\mu N_{sd}}{D} \tag{4.1}$$

where D is the horizontal displacement of the system and N_{sd} is the normal load on the bearing:

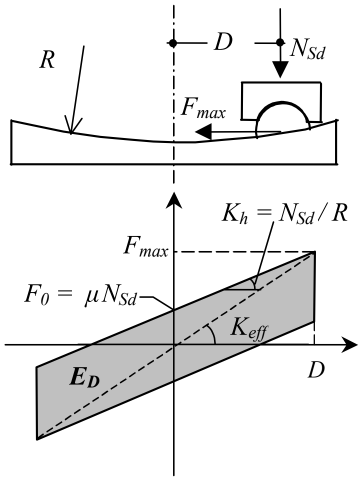
$$N_{sd} = m\,g \tag{4.2}$$

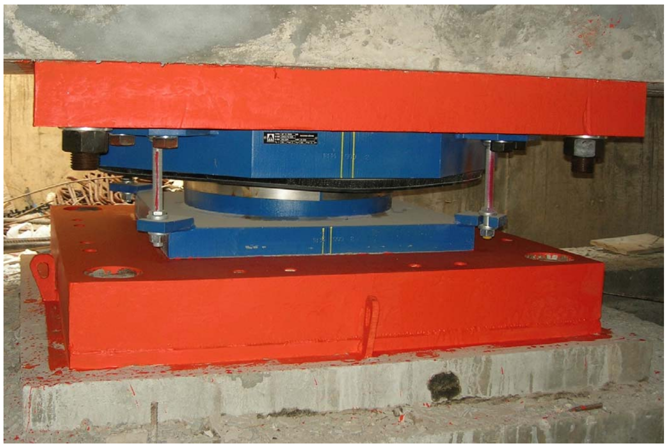
The fundamental period of the system is:

$$T_{eff} = 2\pi \sqrt{\frac{m}{K_{eff}}} \tag{4.3}$$

Eqn. 4.3 after substituting parameters K_{eff} and m from Eqns 4.1 and 4.2 respectively, results to:

$$T_{eff} = 2\pi \sqrt{\frac{R\,D}{g\,D + \mu\,g\,R}} \tag{4.4}$$





Figures 1, 2. Bilinear model of FPS type isolator and picture of the isolator (inverted configuration as placed).

Thus the fundamental period depends on the radius of curvature of the sliding surface, the friction coefficient and the horizontal displacement of the system. Obviously, the stiffer the structure, the more accurate is modeling by a SDOF system.

4.2. Design response spectra

As has been already mentioned, the superstructure was designed for an earthquake with a return period of 475 years which is the design basis earthquake (DBE). It is defined by a PGA of 0.24g, with an importance factor 1.15 and a behavior factor q =1.50, corresponding to elastic behavior. Following the principles of capacity design, the substructure was designed with increased seismic specifications when compared to the superstructure. Thus it was designed for DBE but with a behavior factor of q =1. Following the same principles, the isolation system was designed with even more enhanced seismic specifications, which in this case was the maximum considered earthquake, (MCE) an earthquake with a return period, T_R, of 2400 years. In addition, soil class was taken as B, according to geotechnical investigation. All of the above are reflected on the spectra of Fig. 3. These spectra are based on the elastic spectrum of GSC-2000, modified for the reduction due to the damping of seismic isolation system. This reduction, is a damping correction factor -that multiplies seismic demand- having a value of η =57.5, as suggested by the consulting firm for implementing seismic isolation, Seismomonosis SA. However this value was used only for the spectrum of MCE, while a more conservative value of η=70% was used for the spectra employed for design of both substructure and superstructure. It should be noted that, as shown in the spectra, these reductions were applied for periods, $T > 0.8\ T_{eff}$.

4.3. Design of seismic isolation system

Seismic isolation system was designed so that the structure would have a specific desirable dynamic behavior. The critical dynamic characteristic is the fundamental period; as period increases, spectral acceleration decreases but there is an increase in maximum displacement according to the following equation:

$$SD = \frac{T^2 SA}{4\pi^2} \tag{4.5}$$

where SA and SD are spectral acceleration and spectral displacement respectively. Obviously an increase in the displacement will result to a demand for bigger gap to ensure the free movement of the structure, boosting up the final cost as result of reduced users' space and more expensive construction of ramps, elevators and staircases. Using as upper limit for the displacement the value of 25.5 cm, then for the MCE the fundamental period of the structure should be around 2.35s. The corresponding spectral acceleration is SA=0.132 g (Fig. 3)

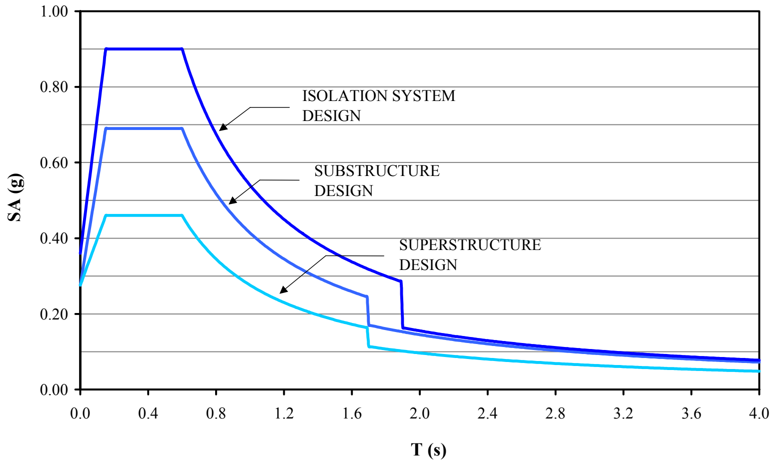


Figure 3. Acceleration response spectra for the design of superstructure, substructure and isolation system.

while the spectral displacement in direction X or Y, coming from Eqn 4.5 or alternatively from the spectra of Fig. 6 is Dx_{2400}=Dy_{2400}=18.1 cm. The final displacement comes from the combination of the horizontal components and lies between D_{2400}=18.9 cm, if seismic force in one direction is combined with 30% of the seismic force in the orthogonal direction, to D_{2400} = 25.6 cm if SRSS rule is used.

Friction coefficient, μ, of the isolators can vary from 0.050 to 0.066 (according to the specifications provided by Seismomonosis SA). The smaller value, μ=0.050, corresponding to a bigger fundamental period according to Eqn 4.4, is obviously critical for the horizontal displacement of the building. For this value, the radius of curvature of the isolator, R, as can be derived from Eqn 4.4, is 2.24 m, thus from Eqn 4.3, K_{eff} = 221,917 KN/m. For DBE, μ is taken as 0.066 since this is the critical value for spectral acceleration and consequently for seismic base shear. Solving the system of Eqns 4.4 and 4.5 for R=2.24 m, as found previously, the values of Dx_{475}=Dy_{475}=15.5cm and T=2.15s are found regarding maximum displacement and fundamental period respectively. Then from Eqn 4.3, K_{eff} =270,177 KN/m. Finally the corresponding spectral acceleration for the superstructure is 0.09g. It should be noted that had the building been designed without seismic isolation then, as concluded from preliminary studies, its periods in the horizontal direction would lie between 0.20-0.34s (on the horizontal part of the spectrum), resulting to a spectral acceleration of 0.46g, which is more than 5 times bigger.

5. DYNAMIC RESPONSE SPECTRUM ANALYSIS ON A 3D MODEL

Dynamic response spectrum (R-S) analysis on a 3D finite element model is the next step of the design methodology aiming to: (a) verify the fundamental period (b) Calculate maximum displacements (c) check for uplift and finally (d) do the final design of structural elements.

5.1. Description of structural system- Structural model

Seismic isolation at the ground floor level separates superstructure from substructure. The load transmission between the two sections is carried out through 46 friction pendulum isolators (SIP, Maurer) placed underneath the thick ground floor slab (1.50 m). The bearings are placed in an inverted configuration (Fig. 2), with the spherical surface on top, so that during seismic movement there will be no additional moments, due to eccentricity, induced to the structural elements that lie underneath (especially the columns). It is the ground floor slab that is designed to account for these additional moments. The structure is based on a mat foundation at -27.70 m from ground level. The structural system of the basement consists of a perimeter shear wall and of wall-beam and column-beam frames in the interior. All vertical structural elements, just below the isolation level, are connected through a horizontal steel truss that functions as a diaphragm in order to eliminate any differential displacements during a seismic event. Superstructure consists of two parts. The main feature of the first is the oval-shaped shell structure supporting the slabs of the auditorium balconies. The shell is surrounded

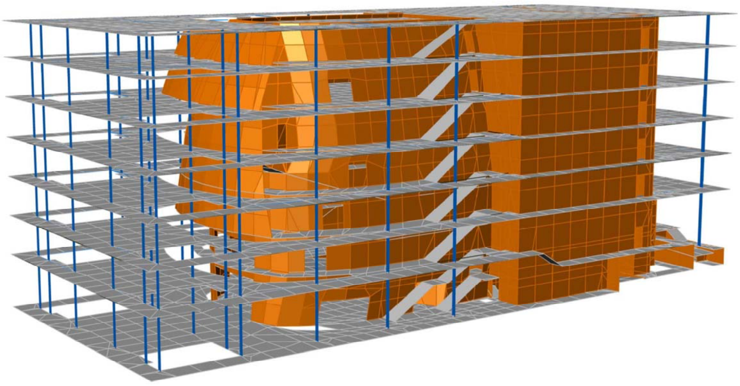


Figure 4. Structural model of superstructure.

by slender column-beam frames. The second part consists mainly of an extended U shaped shear wall. Analysis was performed through the use of computer code ETABS. Two separate 3D finite element models were created for the superstructure and the basement respectively, in order to increase the accuracy and the ease of calculations. Superstructure was considered elastically supported on the isolators in the vertical direction. Spring values reflect both soil and substructure's stiffness. In the horizontal direction, the total effective stiffness was distributed to each bearing proportionally to the normal load it carries.

For the analyses, the response spectra presented in Fig. 3 were used, corresponding to the design of: (a) the isolation system, (b) the substructure and (c) the superstructure. From spectrum (a) displacements for MCE are checked and the bearings were designed. Spectrum (b) was used to design the substructure and calculate displacements for DBE. Finally spectrum (c) was used to design the superstructure. Regarding the vertical component of the excitation, spectra (a) and (b) were used for MCE and DBE respectively, after being multiplied by 0.70 according to GSC-2000.

5.3. Results from Dynamic Response Spectrum Analysis, design basis earthquake level

Response spectrum analysis of the model for DBE, results in two translational modes of vibration which have fundamental periods of 2.15s and 2.17s for directions X and Y respectively and stimulate the entire mass of the structure. In the vertical direction, the structure is rather stiff and large number of modes is required in order to achieve a total participation factor 100%. Periods found lie in the interval of 0.10-0.30s. Using response spectra (b) and (c,) structural elements of the basement and the superstructure, respectively, were designed. In table 2 dimensioning of a typical circular column at top floor is presented and compared to the one of the fixed based model of the structure. In Table 2 story drifts, normalized with respect to storey height, $\gamma = \Delta / h$, in directions X and Y are presented. Comparing these values to the ones obtained for a conventional structure shows substantial reduction. It should be noted that although the values calculated for the conventional structure are below $\gamma = 5\times10^{-3}$ that is usually considered as a limit for damages to non-structural elements, these are critical values for the marble and glass facades of the building. Horizontal displacement found is Dx_{475}=Dy_{475}=15.5cm. The final displacement comes from the combination of the horizontal components and lies between D_{475}=16.2cm, if seismic force in one direction is combined with 30% of the seismic force in the orthogonal direction, to D_{475}=21.9 cm if SRSS rule is used. Finally there is no uplift present.

Table 1. Design of a circular column at top floor, for different models and analysis methods, DBE.

Model / Analysis method	Diameter (cm)	Reinf (cm^2)
Fixed base / R-S analysis	Ø75	73
Base Isolated / R-S analysis	Ø65	47
Base Isolated / t-h analysis El Centro	Ø45	24
Base Isolated / t-h analysis L.California	Ø45	40

Table 2. Comparison of normalized story drift, $\gamma = \Delta / h$, for different models and types of analysis.

Storey	Height	Conventional structure R-S analysis		Seimic isolated structure R-S analysis		Seimic isolated structure Nonlinear t-h analysis	
	H (m)	$\gamma_{X,max}$ (10^{-3})	$\gamma_{Y,max}$ (10^{-3})	$\gamma_{X,max}$ (10^{-3})	$\gamma_{Y,max}$ (10^{-3})	$\gamma_{X,max}$ (10^{-3})	$\gamma_{Y,max}$ (10^{-3})
7	26.53	0.63	1.13	0.22	0.31	0.20	0.29
6	22.87	0.93	1.44	0.29	0.34	0.17	0.34
5	19.20	0.83	1.85	0.31	0.40	0.19	0.39
4	15.53	0.85	1.85	0.31	0.45	0.19	0.42
3	11.87	0.82	2.11	0.32	0.40	0.15	0.39
2	8.20	1.00	1.76	0.32	0.64	0.17	0.60
1	3.20	0.63	3.01	0.25	0.38	0.17	0.28
0	0.00						

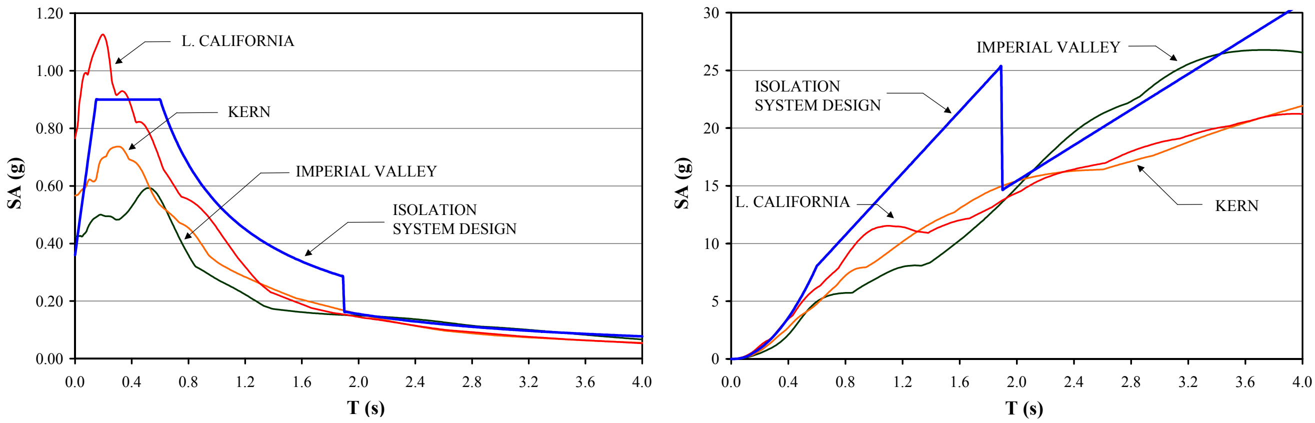
Response spectrum analysis of the model for MCE, results similarly in two translational modes of vibration that have fundamental periods of 2.35s and 2.38s for directions X and Y respectively and stimulate the entire mass of the structure. In the vertical direction, the modes and corresponding periods are the same with those found for DBE (same vertical spring values). Horizontal displacement at the isolation level is Dx_{2400}=Dy_{2400}=18.2cm. The final displacement comes from the combination of the horizontal components and lies between D_{2400}=19cm, if seismic force in one direction is combined with 30% of the seismic force in the orthogonal direction, to D_{2400}=25.7 cm if SRSS rule is used. Again no uplift is present.

5.5. Conclusions from Response Spectrum Analysis

Fundamental periods found from response spectrum analysis on a 3D finite element model are almost identical to the ones from the equivalent SDOF system calculations. This shows that the structure is not flexible so modeling as SDOF system is valid assumption. As a result, the displacements found, show also good agreement. In addition when checking both for DBE and MCE there is no uplift present. Finally, these results were used to design the structure.

6. NONLINEAR ANALYSIS

To develop additional insight on the performance of the structure but also in order to design the isolation system, nonlinear time-history analyses were performed.



Figures 5, 6. Spectra derived from horizontal components of scaled acceleration and displacement spectra of selected records for the design of the isolation system for Maximum Expected Earthquake (β=25.2%).

Table 3. Scaling factors (SF), DBE / MCE (T_R = 475 / 2400 y).

Accelerogram	El Centro	Kern	Lower California
SF for horizontal component	1.10 / 1.35	2.30 / 3.10	3.00 / 4.10
SF for vertical component	1.05 / 1.15	2.10 / 2.60	2.70 / 3.70

6.1. Model

For this part of the analysis, the 3D finite element model used previously (Fig. 4) was modified after modeling isolators as nonlinear elements in the horizontal direction while in the vertical, the same linear springs as in response spectrum analysis were used. Nonlinear properties differ in regards to the friction coefficient, depending on the seismic demand (DBE or MCE).

6.2. Accelerograms

For nonlinear time-history analysis, three events (Table 3) that correspond to the seismic profile of the region regarding soil conditions (stiff soil), fault distance and magnitude of event were selected: El Centro (Imperial Valley EQ, May 18, 1940, station #117), Kern (Kern County, California EQ, Jul. 21, 1952, station #095) and Lower California (Lower California EQ, Dec. 30, 1934, station # 117). Pairs of horizontal ground motion t-h components were scaled and the SRSS of the 5% damped spectrum of the scaled, horizontal components divided by 1.3 (derived spectra) was constructed. The motions were scaled such that the average value of the derived spectra does not fall below the spectrum of the DBE (or MCE) by more than 10% for periods from $0.5T_{eff}$ to $1.25T_{eff}$. The vertical components were also scaled. Scaling factors, for both levels of seismic design, for El Centro, Kern and Lower California records are tabulated in Table 3 while modified spectra for accelerations and displacements are presented in Figs 5 and 6 respectively. Scaled accelerograms in the three directions were applied simultaneously and 8 combinations of seismic action were examined per earthquake and level of seismic action (a total of 48 combinations).

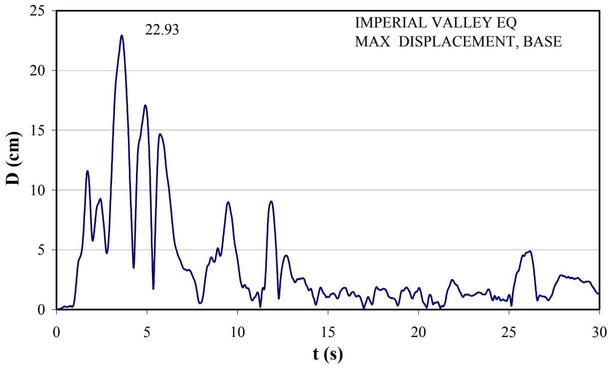
6.3. Time-history analysis results

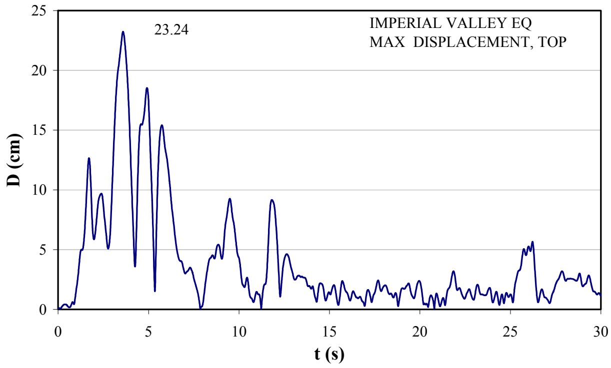
For DBE analysis focused on section forces on structural elements, that from the previously performed response spectrum analysis, were considered critical. These include the columns of the basement, just below the isolation level, and the steel truss that connects them. Results were also used to check the design of several other elements (for instance the circular column at top floor presented in Table 1). Finally, Table 2 is completed by filling in the normalized storey drifts of the structure.

For MCE analysis, results include section forces on bearings and displacements. Maximum calculated displacements are 21.1cm, 17.0cm and 18.4cm for El Centro, Kern and Lower California earthquakes respectively resulting to an average displacement value of 18.8 cm. In Figs 7 and 8, displacement t-h at the ground and top floor level respectively is presented. It is apparent that storey drift is very small.

6.4. Conclusions from time-history analysis

Results showed that scaled El Centro records were critical for displacements, as could have been predicted from displacement spectra of Fig. 6, while scaled Lower California records were critical for section forces on structural elements. In regards to displacements there was good agreement between results from nonlinear t-h analyses to those from R-S analyses. However this is not the case regarding section forces and this is apparent in the design of the circular column presented in Table 1. This can be attributed to the way vertical component of excitation is combined with horizontal components in response spectrum analysis and results to overdesign (with the exception of a structure near a fault). This may also suggest that if in a similar project R-S analysis results show that there is uplift this should be reexamined after t-h analyses are performed.





Figures 7, 8. Displacement t-h at the ground and top floor level respectively, MCE (El Centro, 1940).

7. CONCLUSIONS

Dynamic behavior of Onassis House of Letters and Arts was presented. The study focused on the methodology followed for the seismic design which, performed in three consecutive steps, included simple calculations using an equivalent SDOF system, dynamic response spectrum analyses and non-linear time-history analyses using selected earthquake records on a 3-D finite element model. The following conclusions were derived:
a. From the simplest approach -SDOF system- to the more sophisticated type of analysis -nonlinear time-history on a 3D finite element model- results (periods, displacements) show good agreement. This can be attributed to the stiffness of the structure.
b. In regards to section forces, results obtained from response spectrum analysis are more conservative than those obtained from time-history analysis. This is attributed to the contribution of the vertical seismic component in the envelope of imposed actions in response spectrum analysis.
c. Had the building been designed without seismic isolation the story drifts would be unacceptable and the structural system would not be as elegant as that required by the architectural concept.
d. In addition, the incorporation of the isolation system leads to substantially lower design acceleration. A conventional structure would have to be designed for acceleration five times greater. Similarly, the contents of the building would experience significantly greater accelerations in a conventional structure.

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